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DURABILITY OF MARINE CONCRETE UNDER NEW ZEALAND CONDITIONS: ACTUAL & MODELLED PERFORMANCE AFTER 13 YEARS' EXPOSURE

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SUMMARY

Concrete made using only GP Portland cement is too permeable to chloride ions to adequately protect mild steel reinforcement at conventional covers in aggressive marine environments. This is recognised by the prescriptive durability design solutions in Section 3 of NZS 3101:2006. These solutions mandate the use of supplementary cementitious materials (SCMs) for the most severe C Zone Exposure Classification, in addition to placing restrictions on water to binder ratio and binder content.

Anticipating a need for data on the performance of SCM concretes, experimental exposure sites were established in 1998 representing both C Zone (direct wave splash) and B2 Zone (wind-blown chloride aerosol) environments. The initial concrete specimens exposed included mixes containing blast furnace slag, silica fume and a natural silica pozzolan, plus pure GP cement controls. Later, fly-ash concretes were also installed.

This paper presents the results of recent sampling to measure the performance of the concrete after 13 years' exposure. This information is helpful to validate the current NZS 3101 prescriptive requirements, which were primarily obtained by averaging results from multiple life prediction models using default values derived from overseas data. It also provides a data set based on the performance of well-characterised concrete under local conditions, to serve as robust input to models in situations where alternative durability solutions are desired.

The results are presented as chloride profile vs. depth profiles, which are characterised by surface chloride concentrations and diffusion coefficients, the parameters routinely used for durability modelling. The evolution of the profiles is considered and compared to both NZS 3101 requirements and to the assumptions inherent in some of the more accessible service life prediction models.

INTRODUCTION

The durability of concrete structures in marine environments is largely governed by the length of time necessary for chloride ions from the surroundings to penetrate the cover concrete and accumulate at the depth of the reinforcement in sufficient concentration to initiate corrosion.

Since the mid-1990s, it has been considered best-practice amongst concrete technologists to model chloride ingress by applying a variation on the error function solution to Fick's 2nd law of diffusion derived by Crank (1975). Crank's original equation is modified such that the diffusivity of the concrete (i.e. its resistance to chloride ions) decreases with time according to relationships with the typical form (Frederiksen et al., 2009):

$$D_{av,t} = D_{ref}\left(\frac{t}{t_{ref}}\right)^{-\alpha} \quad [1]$$

where $D_{av,t}$ is average chloride diffusion coefficient of the concrete over the time interval t since first exposure, D_{ref} is a reference measurement of diffusivity made at some earlier time t_{ref}, and the age exponent α is a constant that depends on the composition of concrete and its environment (refer to Listing 1 for the definition of other common abbreviations used in the text). Popular service life prediction models based on using Fick's law with a reducing diffusion coefficient include '*LightCon'* (Maage et al., 1995), *'DuraCrete'* (1998), the UK Concrete Society's *'AgedDCA'* (Bamforth, 2004) and '*Life-365*' (Ehlen et al., 2009), which is developed and maintained by a US concrete industry consortium including the National Ready Mixed Concrete Association.

The use of such models is permitted by the *'Design for Durability'* chapter of the New Zealand *Concrete Structures Standard*, NZS 3101:2006, as an alternative to the prescriptive tables of concrete quality and minimum covers necessary to obtain mandated service life requirements. However, the covers chosen for the prescriptive solutions for the B2 and C exposure zones themselves were also strongly influenced by life prediction modelling to demonstrate their adequacy.

Prediction models based on Fick's Law do not aim to accurately simulate the physical and chemical processes involved with chloride ingress into concrete, which is presently an ideal that is not within practical reach. Instead their ambition is to describe the results of these processes, i.e. the chloride concentration vs. depth profiles developed in the concrete with time. The accurate application of these models is therefore dependent on careful calibration against good measurements from both the environment and concrete type under consideration. This is especially true for the value of the age exponent α, which is the dominant influence on the resistance of concrete to chloride ion penetration over its service life.

In recognition of the need for calibration data for locally-used cementitious materials in New Zealand environments, BRANZ instigated a programme monitoring the performance of concrete exposure blocks in marine environments, commencing in late 1998. This paper highlights some of the results of the most recent sampling and analysis of the blocks after 13 years' exposure. A full description of all the data collected will be published as a freely available Study Report, accessible from the BRANZ website (www.branz.co.nz).

α	Regression parameter describing the rate at which the achieved chloride diffusion coefficient of the concrete is decreasing with time, *non-dimensional*.
$C_{(x,t)}$	Chloride content of the concrete at depth *x* and time *t, %w/w of binder*.
C_i	Initial chloride content of the concrete prior to exposure, *%w/w of binder*.
C_{cr}	Critical corrosion threshold: the minimum chloride concentration necessary to initiate reinforcement corrosion, often conservatively assumed to be 0.4% by mass of binder for mild steel reinforcement, *%w/w of binder*.
C_{sa}	Regression parameter obtained by fitting experimental data for $C_{(x,t)}$ to an idealised profile. It describes the chloride content of the infinitely thin depth increment of concrete immediately below the exposed surface, *%w/w of binder*.
$D_{(t)}$	Instantaneous diffusion coefficient at time *t*, $mm^2/year$.
D_{ach}	'Achieved' (apparent) diffusion coefficient over a time interval, obtained as a regression parameter when fitting experimental data for $C_{(x,t)}$ from a measured chloride profile to Crank's solution using the erfc function, $mm^2/year$.
$D_{av,\Psi}$	Time-averaged diffusion coefficient obtained as a regression parameter when fitting experimental data for $C_{(x,t)}$ to Mejlbro solution using the psi (Ψ) function, $mm^2/year$.
$D_{av,ref}$	Reference achieved chloride diffusion coefficient, measured at some time t_{ref}, mm^2/yr.
$\mathrm{erfc}(u)$	Gaussian complementary error function, *non-dimensional*.
p	Regression parameter describing the rate at which C_{sa} increases with time, *non-dimensional*.
t	Age of concrete after casting, *years*.
t_{ex}	Age of concrete at first exposure to chlorides; generally neglected if $t>>t_{ex}$, *years*.
S_{ref}	Reference parameter describing the value of C_{sa} at some t_{ref}, *% w/w of binder*.
x	Distance below the exposed concrete surface, *mm*.
$\Psi_p(z)$	Mejlbro psi function as defined in the text, evaluated where C_{sa} is a temporal variable whose rate of change is defined by *p*.

Listing 1. Definitions for abbreviations & mathematical symbols used in the text.

EXPOSURE SITE PROGRAMME

Two exposure sites were established in the Wellington region, one at Weka Bay (at map coordinates 41.295 S, 174.805 E) in the inner harbour and the second at Oteranga Bay (41.295 S, 174.629 E) on the exposed southern end of Cape Terawhiti, 16 km west of the city. The concrete samples at Weka Bay are situated above the spring high tide line but are regularly wetted by waves on breezy days and the site is considered to be a fairly aggressive example of an NZS 3101 C zone exposure classification. At Oteranga Bay, the samples are situated within 100 mm of the shoreline and subject to high rates of salt deposition from wind-blown aerosols. As such, the site is considered to be a severe example of the B2 coastal frontage zone under NZS 3101.

Four series of mixes were initially produced, consisting of a control series containing only type GP cement and three further series in which a quantity of the binder was replaced with three common SCMs then available on the New Zealand market: *Duracem* ground granulated blastfurnace slag cement, *Micropoz* silica fume and *Microsilica 600* natural silica pozzolan. Each series included mixes at three different levels of total cementitious material: 280 kg/m^3, 325 kg/m^3 and 400 kg/m^3, as appropriate for the severity of the exposure environment for which they were intended. The mix designs were intended to be representative of commercial ready-mixed concrete and were wet-cured for 7 days prior to transport to the exposure sites approximately one month after casting.

In 2004, the original mixes were supplemented with a new series of blocks an additional series of concretes were produced, using two Class C and a Class F fly ash as partial cement replacements, reflecting the growing popularity of this variety of SCM. In accordance with BRANZ' confidentiality agreement with the fly ash suppliers, the specific brand of each fly ash is not disclosed. Due to a combination of technical considerations and budgetary constraints, the later fly ash mixes were produced with a total binder content of 370 kg/m^3 and located on the C zone exposure site only. Table 1 summarises the concrete types included in the study.

Table 1. Concrete mix types included in the study.

Mix Code	Binder / SCM	Description	Cement Replacement	Exposure Date
GP	Golden Bay GP	Type GP Portland cement	Nil	Dec 1998
DC	Duracem	Blast-furnace slag cement	50 – 65%	Dec 1998
MP	Micropoz	Silica fume	5 – 8%	Dec 1998
MS	Microsilica 600	Natural amorphous silica	5 – 8%	Dec 1998
FA-C1 & C2	Fly ashes	Type C fly ash (2 suppliers)	30%	May 2004
FAF	Fly ash	Type F fly ash	30%	May 2004

RESULTS & ANALYSIS

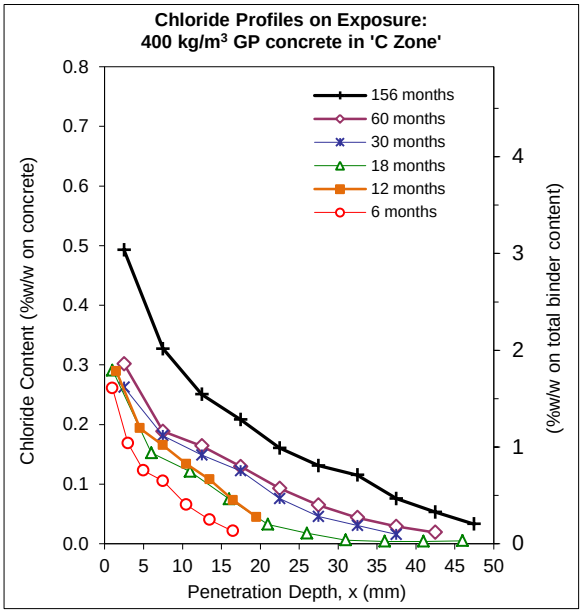
Crank's error function equation [2], the basis of many service life prediction models, is a direct solution to the initial / boundary value problem stated in [3].

$$C_{(x,t)} = C_i + (C_{sa} - C_i) \cdot \operatorname{erfc}\left(\frac{x}{2\sqrt{D_{(t)} \cdot (t - t_{ex})}}\right) \quad [2]$$

$$[3] \quad \begin{cases} \frac{\partial C}{\partial t} = D_{(t)} \frac{\partial^2 C}{\partial x^2} & x > 0,\ t > t_{ex} \quad (\text{Fick's } 2^{\text{nd}} \text{ Law}) \\ C_{(0,t)} = C_{sa} & t > t_{ex} \quad (\text{boundary condition}) \\ C_{(x,t_{ex})} = C_i & x > 0 \quad (\text{initial condition}) \end{cases}$$

The boundary condition that must necessarily be imposed in [3] in order to obtain a unique solution to the partial differential equation that expresses Fick's 2nd law ***requires*** that the surface chloride value (C_{sa}) is invariant, or at least establishes a constant value very quickly. The most surprising finding from the most recent inspection data is that this requirement is not necessarily obeyed by the samples, particularly for the concrete exposed on the C zone exposure site.

This is illustrated by Figure 1, which shows two examples of the chloride concentration vs depth profiles developed within two of the 400 kg/m^3 binder concrete specimens exposed on the C zone exposure site. It is clear in both cases that the surface chloride concentration C_{sa} (i.e. the extrapolation of the idealised profile back to zero depth) has increased substantially between 5 and 13 years. It is also clear that while a qualitative judgement can readily be made that the GP control mix is less resistant to chloride penetration than the concrete containing Microsilica, it is less obvious what implications this would have for the intended service life of a structure without an accurate method of predicting the future evolution of the profiles.



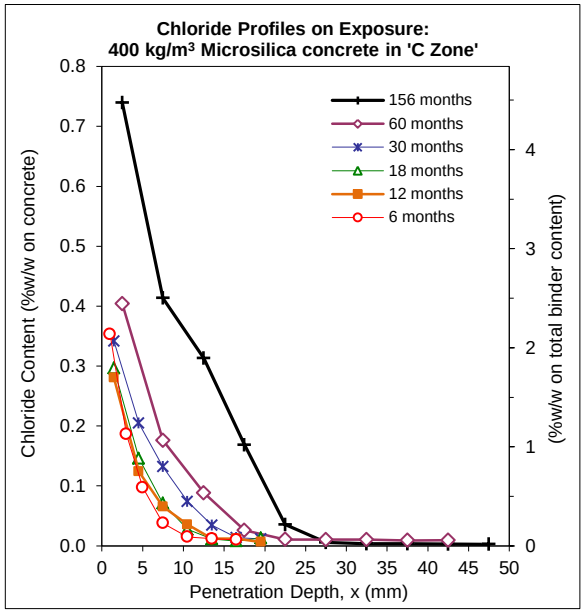


Figure 1. Example chloride profiles for two 400 kg/m^3 total binder content concrete mixes exposed on the Weka Bay C zone site.

Modelling these profiles with the assumption of a constant rather than increasing C_{sa} using equation [2] risks underestimating the average diffusivity of the concrete, which in-turn leads to an overly-optimistic assessment of service life.

A review of the literature reveals that Mejlbro (1995) provided a generalised solution to Fick's 2nd law that allows for both a time-dependent diffusion coefficient and surface chloride concentration. Mejlbro considered the case in which C_{sa} is allowed to vary in accordance with a power function of time, as suggested by Swamy et al. (1994) on the basis of observations of existing concrete structures. The surface chloride concentration is explicitly linked to the concrete type and exposure environment by incorporating the value of the chloride ion diffusion coefficient:

$$C_{(x=0,t)} = C_{sa} = S \cdot \{(t - t_{ex})D_{(t)}\}^{p} \quad [4]$$

Under this alternative boundary condition, the solution to Fick's 2nd law (Poulsen & Mejlbro, 2006) is given by:

$$C_{(x,t)} = C_i + (C_{sa} - C_i) \cdot \psi_{\mathrm{p}}(u) \quad [5]$$

where

$$C_{sa} = S_{ref}\left(\frac{t}{t_{ref}}\right)^{(1-\alpha)p} \quad [6] \qquad \text{and} \qquad u = \frac{x}{2\sqrt{t \cdot D_{av,\Psi(t_{ref})} \cdot \left(\frac{t}{t_{ref}}\right)^{-\alpha}}} \quad [7] \quad \text{for} \quad t, t_{ref} \gg t_{ex}$$

Consequently the chloride accumulation in the concrete $C_{(x,t)}$ at any depth x and time t can be completely specified providing four parameters are known: α, p, $D_{av,\Psi(tref)}$ & S_{ref}. The values of α and p describe the rate of change with time of D_{av} and C_{sa} respectively and the other two parameters are point-wise measures of these properties after some reference period of exposure.

The value of the Ψ function for any given p & z is evaluated by the expression:

$$\psi_{\mathrm{p}}(\mathrm{z}) = \sum_{n=0}^{+\infty} \frac{p^{(n)}(2z)^{2n}}{(2n)!} - \frac{\Gamma(p+1)}{\Gamma(p+0.5)} \cdot \sum_{n=0}^{+\infty} \frac{(p-0.5)^{(n)}(2z)^{2n+1}}{(2n+1)!} \quad [8]$$

where $\Gamma(y)$ is the Gamma function defined by:

$$\Gamma(y) := \int_0^{+\infty} u^{y-1} e^{-u} du \quad [9]$$

and $p^{(n)}$ is the 'falling factorial' defined as:

$$p^{(0)} = 1;\ p^{(1)} = p;\ p^{(2)} = p(p-1);\ \ldots;\ p^{(n)} = p(p-1)(p-2)\cdots(p-n+1);\ \text{[10]}$$

The difficulty of applying Mejlbro's solution lies in the novelty of the Ψ (psi) function, which effectively provides a generalisation of the more familiar complementary error function (*erfc*) to take account of the variable driving force for the diffusion imposed by a changing C_{sa}. For the special case of p=0, which denotes a surface chloride value that remains constant with time, $\Psi_0(z) = erfc(z)$ and Crank's equation [2] is obtained as expected. Unlike *erfc*, Ψ is not an intrinsic function of numerical software, possibly a contributor to Frederiksen et al. (2009) describing this result as "*little known*" amongst concrete technologists.

Irrespective of whether Crank's or Mejlbro's solution is used to model chloride profiles developed in exposed concrete, the value of the diffusion coefficient calculated is not an instantaneous variable that captures concrete performance at the point of inspection. Instead it represents the time-averaged value of the concrete's resistance to chloride ingress since the time of first exposure, i.e.

$$D_{\mathrm{av}}(t) = \frac{1}{t-t_{ex}}\int_{t_{ex}}^{t} D(\tau)\mathrm{d}\tau \quad \text{[11]}$$

It is important to note that the best fit values of average diffusivity D_{av} obtained from fitting each solution to experimental chloride profiles are not identical. An estimation of D_{av} that has erroneously assumed a constant C_{sa} over time will generate errors when the exposure data is extrapolated into the future.

As an example, Figure 2 compares the predicted development of the chloride profiles for the 400 MP (400 kg/m^3 Micropoz) concrete sample on the Weka Bay C zone site with time under two scenarios: The first is that the current (13 year) best-fit surface chloride concentration (4.55% by mass of binder) has been constant and will remain so in the future, while the diffusivity reduces according to the best-fit age parameter α = -0.41. This represents the conventional *erfc* model, which is equivalent to the case where Ψ_p = 0. The second scenario assumes the Mejlbro Ψ_p model is accurate and that C_{sa} follows the boundary conditions given by [5] with p = 0.47 and the diffusivity reducing with age as before. For a nominal cover depth of 50 mm, assumption of a constant C_{sa} indicates a theoretical service life of 100 years before the conventional threshold for mild steel corrosion of 0.4% w/w chloride by mass of binder is exceeded. In contrast, the Ψ_p model predicts this threshold is breached after only 60 years.

Frederiksen et al. (2009) have demonstrated by both numerical methods and pure analysis that there is a relationship between the two estimates for D_{av} that depends only on p and is independent of time, according to the equation:

$$D_{\mathrm{ach,erfc}}(t) = \frac{1}{{\mu_p}^2} D_{\mathrm{av},\psi}(t) \quad \text{[12]}$$

The value of the adjustment factor μ_p is given by the approximating polynominal:

$$\mu_p = 1 + 0.5194p - 0.0876p^2 + 0.0185p^3 - 0.0022p^4 \quad \text{[13]}$$

The relationship between the temporal dependence displayed by each estimation of diffusivity is remarkably simple however, with:

$$\alpha_{\mathrm{erfc}} = \alpha_{\psi} \quad \text{[14]}$$

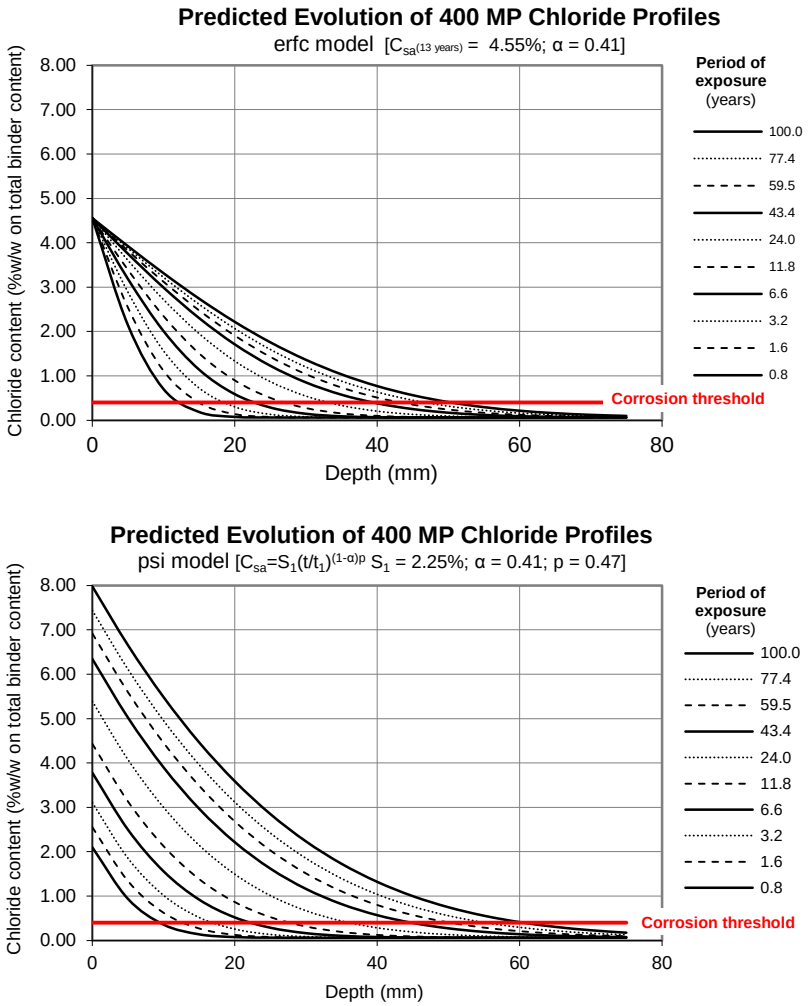


Figure 2. Evolution of the predicted chloride profiles with time for the 400 MP concrete, assuming the conventional *erfc* model applies (top) compared with the result predicted from the Ψ_p model. The *erfc* solution is calculated with the C_{sa} value obtained from the most recent inspection after 13 years' exposure.

The results of using Mejlbro's Fick's law solution to parameterise the chloride ingress profiles over the last 13 years for the concrete mixes on each exposure site are summarised in Table 2. The principal features in this data include:

- The C zone concretes all show a statistically significant reduction in average diffusivity with time, as indicated by the value of the α parameter, and the reduction is greatest for the slag and fly-ash concretes. This trend is present but less certain for the B2 zone concrete where the reduction is smaller and there is greater scatter in the data as demonstrated by the very large uncertainties at the 90% confidence level.

- The C_{sa} values are very clearly a function of binder type, with concretes containing SCMs developing surface chloride concentrations in excess of double the comparable values for the GP control concretes. This difference is primarily reflected in the S_1 parameter (i.e. the surface chloride accumulation after year 1), rather than the power exponent for the rate of increase, $(1-\alpha)\cdot p$, which is broadly similar for all the concrete exposed on the C zone site.

- Only the 325 kg/m^3 concretes on the B2 site have surface chloride concentrations that can be approximated by a broadly constant value, as signified by their generally low values for p in Table 2.

Table 2. Parameterisation of C & B2 Zone chloride profiles in accordance with the Mejlbro Ψ_p solution to Fick's 2nd law with a variable surface chloride concentration.

Concrete Type	Surface Chloride Parameters			Achieved Diffusion Parameters		Derived Values	
	$C_{s\ (current)}$	$S_{(t=1year)}$ (% binder)	(1-α)p	$D_{ach\ (t=\ 1\ year)}$ (mm^2/year)	α (± 90% C.L.)	p	$D_{av,\Psi\ (t=\ 1\ year)}$ (mm^2/year)
325 kg/m^3 mixes – C Zone							
Duracem	9.05	4.29	0.25	35	0.71 ± 0.13	0.85	67
GP	3.33	1.89	0.18	106	0.36 ± 0.21	0.28	137
Micropoz	5.55	2.63	0.29	24	0.22 ± 0.15	0.38	33
Microsilica	7.38	2.33	0.41	44	0.44 ± 0.17	0.74	80
400 kg/m^3 mixes – C Zone							
Duracem	5.42	3.29	0.18	26	0.58 ± 0.14	0.44	39
GP	2.98	1.65	0.20	59	0.30 ± 0.20	0.28	77
Micropoz	4.66	2.25	0.28	27	0.41 ± 0.15	0.47	40
Microsilica	5.21	2.18	0.31	11	0.29 ± 0.08	0.44	17
370 kg/m^3 mixes – C Zone							
Type C fly-ash 1	5.42	2.46	0.39	24	0.90 ± 0.19	3.89	124
Type C fly-ash 2	3.51	2.03	0.27	59	0.59 ± 0.19	0.66	101
Type F fly-ash	6.24	2.54	0.44	25	0.75 ± 0.19	1.79	75
280 kg/m^3 mixes – B2 Zone							
Duracem	2.46	1.29	0.29	24	0.28 ± 0.24	0.40	35
GP	0.90	0.66	0.08	65	0.43 ± 0.29	0.14	75
Micropoz	1.56	0.73	0.23	39	0.19 ± 0.19	0.28	50
Microsilica	1.85	0.42	0.56	52	0.43 ± 0.68	0.99	109
325 kg/m^3 mixes – B2 Zone							
Duracem	0.88	0.98	0.000	7	0.26 ± 0.11	0.00	7
GP	0.86	0.52	0.19	20	0.16 ± 0.39	0.23	25
Micropoz	1.07	1.02	0.003	19	0.37 ± 0.16	0.005	19
Microsilica	1.22	0.90	0.07	16	0.07 ± 0.20	0.08	17

LIFE PREDICTION

Using the Table 2 parameters, predictions have been made for the future evolution of the chloride profiles of the exposure site blocks with time. These results are presented in Figure 3 (C zone samples) & Figure 4 (B2 zone), which show the calculated position of the critical corrosion threshold, i.e. the maximum depth below the exposed surface at which the accumulation of chloride ions is sufficient to initiate reinforcement corrosion. This is conservatively assumed to be 0.4% chloride by mass of binder. The data points plotted as open circles represent the actual positions of this threshold at each inspection interval, obtained simply through interpolation of the measured chloride profiles.

As an indication of the suitability of the durability provisions of NZS 3101: 2006, the plots have also been annotated with the specified minimum covers for a 50 and 100 year design life, as stated in Tables 3.6 & 3.7 of the Standard. The specified strength (f'_c) of each concrete mix has been assigned by assuming its 28 day unconfined compressive strength should be considered equivalent to the target mean strength stipulated for a particular concrete grade under table 2.5A of NZS 3104: 2003 *Specification for Concrete Production*.

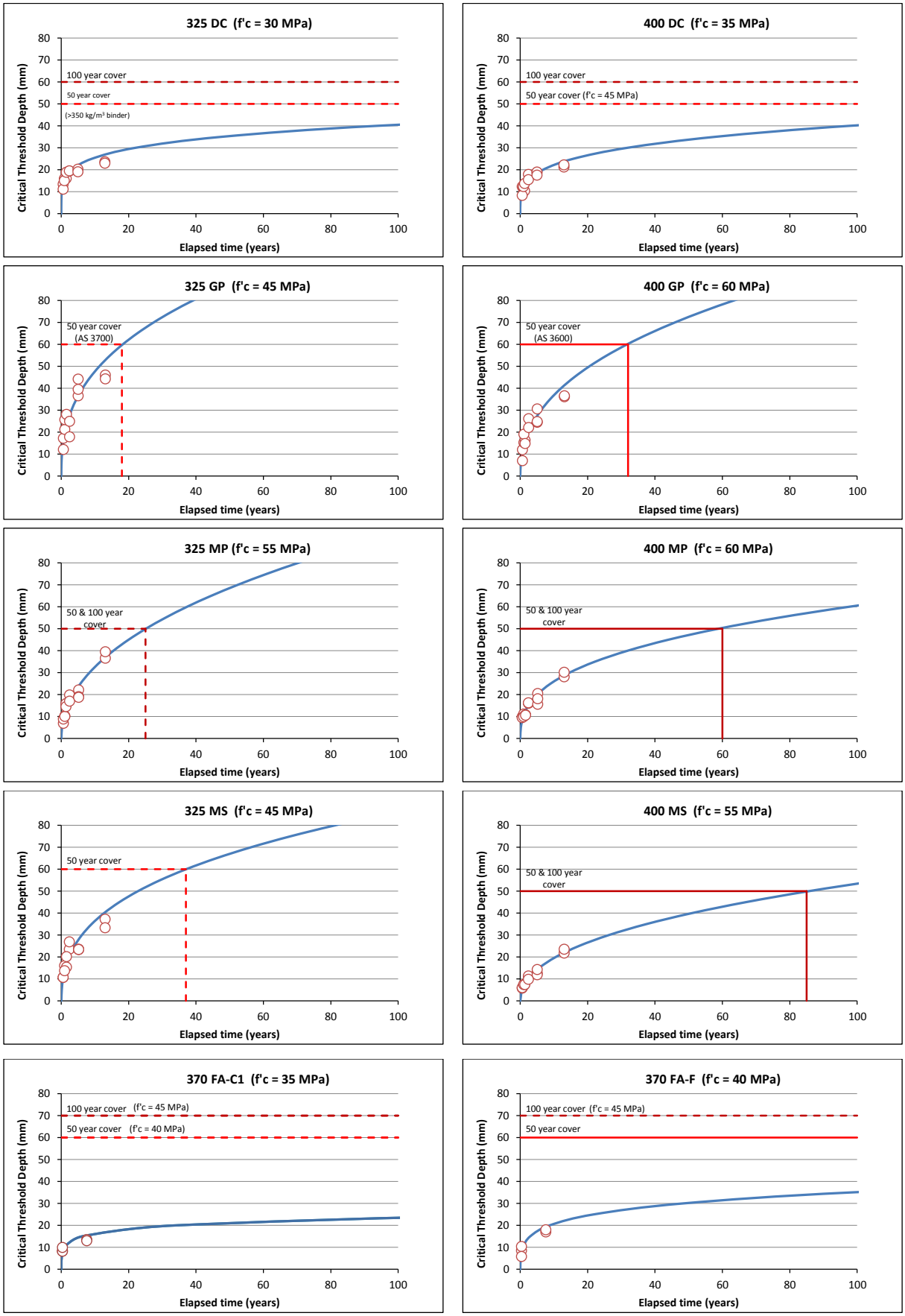


Figure 3. Predicted evolution of the critical corrosion threshold depth (C_{cr} = 0.4% chloride by mass of total binder) with exposure time for the C Zone concretes.

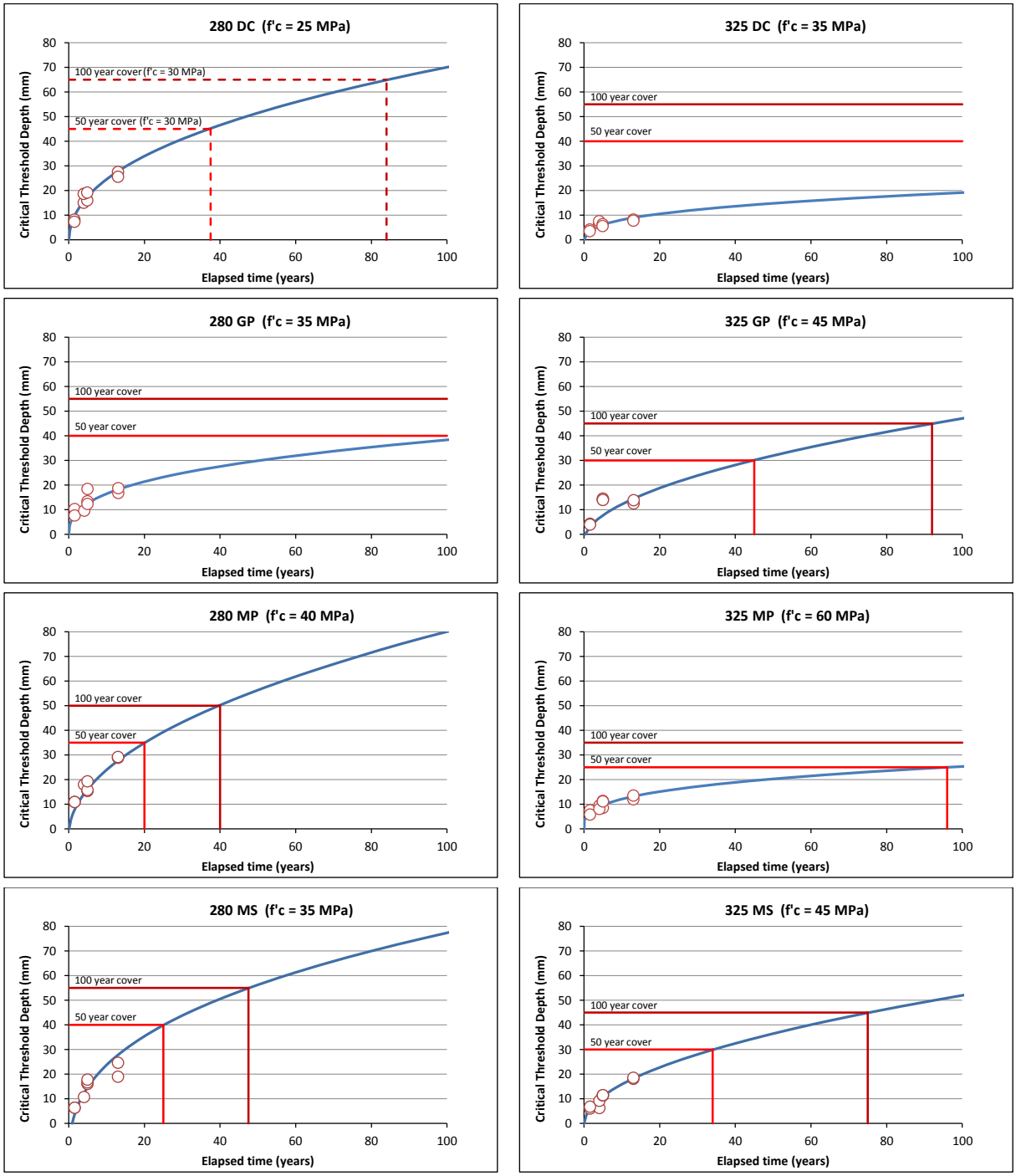


Figure 4. Predicted evolution of the critical corrosion threshold depth (C_{cr} = 0.4% chloride by mass of total binder content) with exposure time for the B2 Zone concretes.

SUMMARY & CONCLUSIONS

Chloride profiles from a series of concrete blocks placed in environments corresponding to the NZS 3101 C and B2 exposure classifications for up to thirteen years have been determined. This data has been fitted to a generalised analytical solution to Fick's 2nd Law of diffusion that takes account of temporal variations in both the surface chloride concentration, C_{sa}, quantifying the stress imposed on the environment and the achieved diffusion coefficient, D_{ach}, characterising the concrete's response to this stress. In conjunction with two further parameters α and p describing the rate at which these properties change with time, the expected surface life of the concretes have been estimated based on a serviceability limit state corresponding to the initiation of reinforcement corrosion. The principal conclusions arising from these findings are:

- For the samples exposed on the C zone exposure site, all the concretes demonstrated a statistically significant reduction in achieved diffusion with time, indicating improved resistance to chloride ingress. The reduction is most pronounced in concrete containing Duracem slag cement or fly ash pozzolans. The dominance of this effect is such that these mixes give longer predicted lives than the silica pozzolan type SCMs included in the trial, despite the fact that their early age diffusivities are comparable.

- To date, the surface chloride concentrations have not stabilised and the value of C_{sa} continues to increase, violating the boundary condition constraint necessary to apply Crank's familiar error function (*erfc*) solution to Ficks' 2nd Law. An accurate model of chloride ingress needs to include such a description of this temporal dependence in the surface chloride. Mejlbro's Ψ_p model, appears to offer a mathematically sound alternative fulfilling this requirement, at the cost of a small increase in computational complexity.

- The value of the time-dependent achieved diffusivity obtained from fitting chloride profiles without also considering the time dependence of the surface chloride concentration is very different to the 'average diffusivity' of the concrete over the period of exposure. For the typical values of p obtained in this study, the true average diffusivity $D_{av,\psi}$ may be 1.5 – 2 times greater than D_{ach}.

- Consequently, time-to-corrosion estimates made with Crank's equation that utilise short-term exposure data are likely to be overly optimistic. Given that this is the basis for a number of popular service life prediction models (e.g. *DuraCrete*, *LightCon* & *AgedDCA*), the validity of their inputs and outputs should be carefully scrutinised before being relied upon for design purpose. Due to the lessening rate of change in D_{av} & C_{sa} with time (the age exponents α & $(1-\alpha)\cdot p$ both being < 1) predictions made with long-term exposure data (e.g. > 30 years) are much less susceptible to this error. Unfortunately this information is frequently only available by inspecting existing structures where the limited ability to accurately characterise the initial concrete quality often restricts the dataset's usefulness for calibrating ingress models.

It should be remembered that the service life predictions in this report are obtained from a conservative definition for end-of-life, suited to design purposes, which negates the duration of the propagation phase between the initiation of rebar corrosion and the development of damage. Moreover, many of the variables treated here as deterministic values have a stochastic component when considered in relation to real structures. This is particularly true of the reinforcement cover distribution and the critical corrosion threshold.

The results from the exposure blocks to date generally support the validity of Tables 3.6 & 3.7 of NZS 3101:2006, which specify the minimum required reinforcement cover and concrete quality for durability design. In particular, it is confirmed that a pure Type GP binder, even in a 60 MPa concrete with a 400 kg/m^3 cement content, is insufficiently resistant to chloride ions to achieve an intended design life of 50 years. Conversely, use of a binder with an SCM is unnecessary for concrete in a B2 exposure classification. Indeed, use of SCMs in concrete with a total binder content of less than 325 kg/m^3 appears to be detrimental to durability performance in this environment, at least on the basis of the chloride ingress observed to date.

While Tables 3.6 & 3.7 are largely fit for purpose in their current from the basis of the mixes considered, the exposure results obtained suggest that the following modifications should be considered for adoption in any future revision of NZS 3101:

- Examine whether the requirements for the use of slag and fly ash concrete in the C zone are overly conservative, in light of the good performance of the apparently non-compliant (below specified strength) mixes tested.

- Increase the minimum cover requirement to 65 mm for the case of a 100 year design life when using concrete with binders incorporating amorphous silica SCMs (e.g. Micropoz silica fume & Microsilica 600) in the C zone.

- For the B2 zone, increase the minimum covers for concrete with a specified strength of $f'_c \geq 45$ MPa by 5 mm where a design life of 50 years is required and by 10 mm for a design life of 100 years.

- Re-evaluate the guidance provided required in the Part 2 Commentary concerning input values for predictive service life models appropriate to New Zealand conditions, particularly in regard to the observed trend of C_{sa} increasing with time over long periods to higher concentrations than previously anticipated. It would also be helpful to try and contextualise this data, allowing it to be usefully generalised to other situations, e.g. where accelerated curing is undertaken in preference to standard wet curing, or for concrete exposed to service temperatures markedly different to those experienced on the BRANZ exposure sites.

Until multi-mechanistic transport models that capture the fundamental physiochemical processes involved with chloride ingress become a practical reality, there is an on-going need for service life prediction models to be founded upon high quality long-term ingress data. It is hoped that the data generated by this research makes a contribution towards fulfilling that requirement.

ACKNOWLEDGEMENTS

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